



## SEISMIC ISOLATION OF THE NUNOA CAPITAL BUILDING, THE TALLEST BASE ISOLATED BUILDING IN THE AMERICAS

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### **Abstract**

This paper describes the selection process for a seismic protection system for the Nunoa Capital Building towers, the tallest isolated residential building in the Americas. A comparison of the seismic performance of these 33-story (42,600 m<sup>2</sup>) towers when protected with viscous walls and a lead rubber bearing base isolation system is presented. The challenges for the designing engineering team and the main design criteria that resulted in the use of 24 large diameter seismic isolators are presented. It is shown that the use of base isolation in this building caused the base overturning moment to increase in comparison to an equal base shear building designed without isolation. Moreover, the requirement of the Chilean isolation code that requires considering a minimum design base shear equal to 5% of the seismic weight, led in this case to design forces almost two times greater than the elastic seismic demand. This code requirement imposed the structural design team two enormous additional challenges: to limit the superstructure interstory drift below 0.25% (code requirement), and to avoid tension and excessive compression forces in the isolation devices, for the design and maximum considered earthquakes, respectively. The interstory drift limit was achieved by using an outrigger system in a non-optimal location due to architectural constraints. Tension forces on the isolators were avoided by interconnecting the slabs of the two towers at the underground levels and by the use of a stiff slab above the isolators. The magnitude of vertical loads on the isolators (up to 40,000 kN) and the need for a long period (nearly 5 seconds) presented some challenges for the design and testing of the isolators. Three types of isolators, with diameters of 115 cm, 135 cm and 155 cm, were used to provide the desired isolated period and damping (20%), while maintaining a vertical frequency in excess of 10 Hz. Isolators were extensively tested at DIS test facility in McCarran, Nevada, USA and at the EU center in Pavia, Italy. Test results validated the properties used in the design and demonstrated the stability of isolators under extreme conditions.

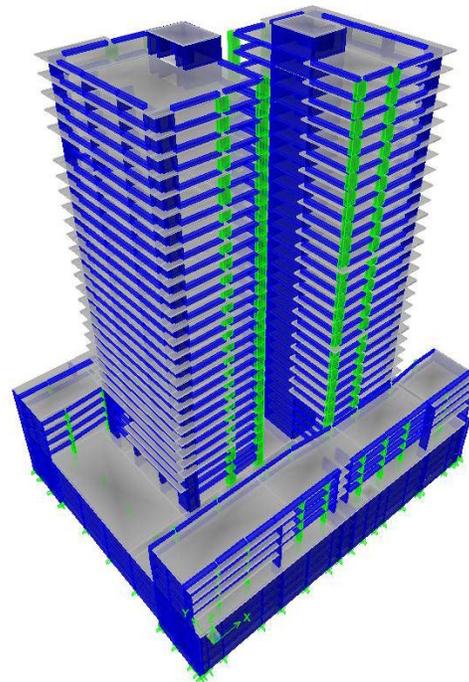
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## 1. Introduction

The Nunoa Capital Building (Fig. 1), with a total constructed area of 42,600 m<sup>2</sup>, consists of two identical towers, both rising 29 floors (75 m) above ground level and both intended for residential use. Around the towers, there are smaller 4 story buildings destined for offices. All the buildings share a common underground level, consisting of four floors. The seismic-resistant system of the building consists of an eccentric reinforced concrete core, as well as a perimeter frame and L-shaped walls in the corners. The floor system is comprised of post-tensioned slabs. A common seismic isolation system has been implemented for the two towers, which are connected at the underground level, and rest on top of a 2 m thick slab supported by 24 natural rubber isolators, 16 of which are lead rubber bearings. The isolators are on footings connected by beams. The isolated towers and the peripheral structures are separated by a 50 cm isolation gap which is significantly larger than the requirement of the Chilean isolation code, NCh2745.Of2003 [1]. The objective of this was to minimize the probability of impact between the isolated structure and the adjacent structures (Fig. 2).

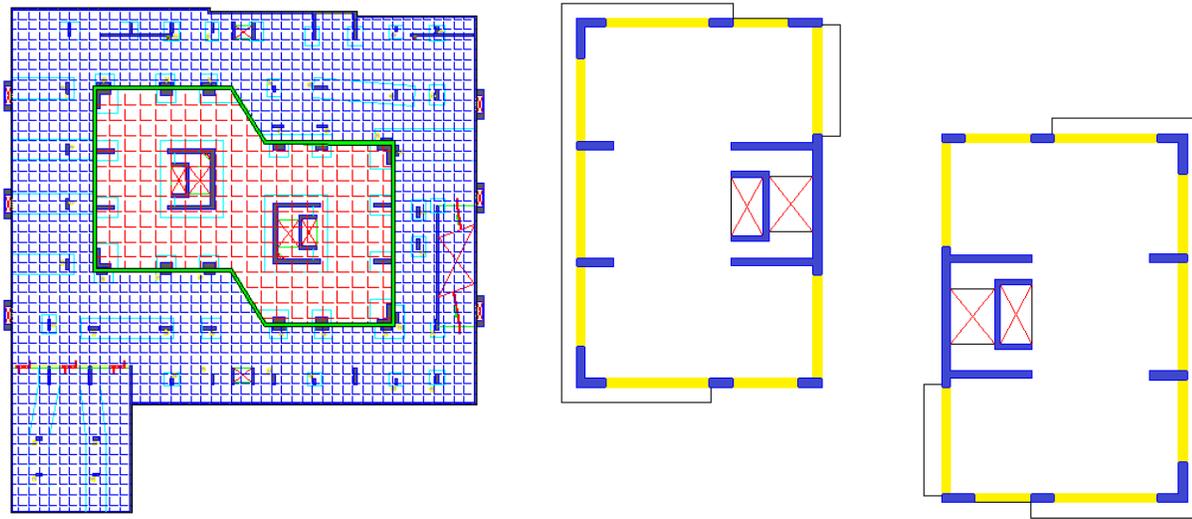


a) Finished Building



b) Analytical Model

Fig. 1 - Nunoa Capital Building



a) Fixed base underground (in blue),  
seismically isolated sector (in red), and  
isolation gap (in green).

b) Typical layout isolated towers.

Fig. 2 - Typical Floor Plan.

For the Nunoa Capital Building, the use of various energy dissipation devices was evaluated. These included viscous dampers, viscous wall dampers, viscoelastic walls, and tuned mass dampers in combination with viscous dampers. The use of viscous dampers was eliminated during the first stages of the project, due to their architectural effect on the facades. For the same reason, the use of tuned mass dampers was also eliminated, as they would only be used in combination with viscous dampers distributed along height. Consequently, in one of the first stages of the project, the use of viscous walls in nonstructural partitions was evaluated, as well as viscous walls coupled with the walls of the elevator shaft. These are shown schematically in Fig. 3. In addition, the feasibility of using a seismic isolation system in two configurations was evaluated: below each tower on the first floor and at the base of the entire structure below the lowest underground level.

Fig. 4 shows a comparison of the seismic responses obtained for the four analyzed cases. This preliminary comparison was completed considering historical earthquake records that are compatible with the spectrum of the Chilean isolation code.

The preliminary analyses completed indicated that the costs of implementing the energy dissipators in partitions and coupling walls was between 2 and 2.5 million US dollars, while the cost of the isolation systems required for isolating the towers individually or together was between 700,000 and 1.2 million US dollars. In light of the economical and technical analyses performed, it was quite evident that the most appropriate option for the structure was the use of seismic isolation systems.

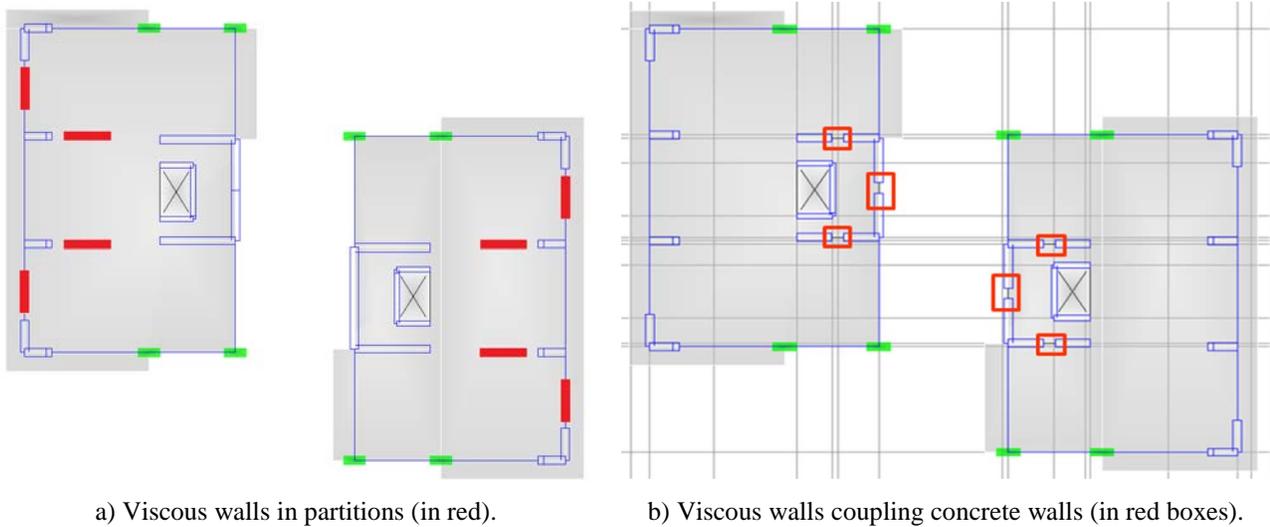


Fig. 3 - Energy dissipation alternatives considered and eventually not selected.

Amongst the existing seismic isolation systems, the team of specialists decided to use a combination of natural rubber and lead rubber bearings, because of the stability and predictability of their properties. Other isolation systems, such as frictional isolators, were discarded due to the difficulties they present for predicting and modeling the variation of the friction in the isolator during seismic movements, given the vertical effects of earthquakes. Similarly, the use of high damping rubber isolators was dismissed for the low level of damping they provide, and for the difficulties in predicting their behavior during severe strong motions.

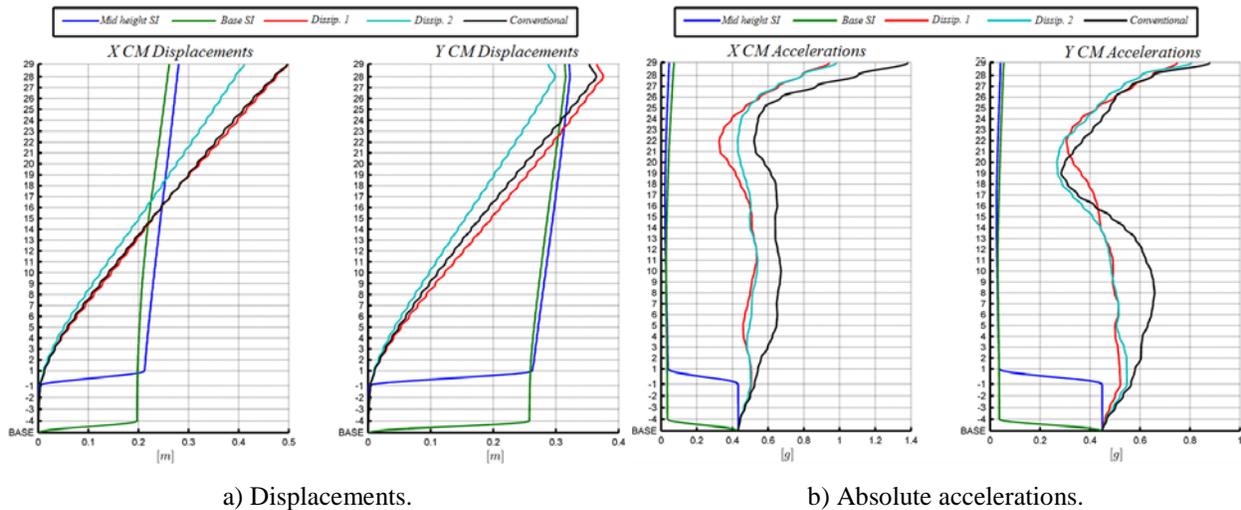


Fig. 4 - Comparison of the elastic seismic response of a conventional structure and the structure with base isolation and energy dissipaters.

Due to the period of the fixed base structure, around 2 seconds, it was estimated that the period of the isolated structure would be around 5 to 6 seconds, and therefore, in accordance with the current Chilean isolation code, a site specific seismic hazard study was required.

## 2. Seismic Hazard Evaluation

Chile is one of the countries with the highest seismic activity in the world, primarily due to the subduction process of the Nazca Plate below the South American continent. This subduction process gives rise to different types of earthquakes, which are classified under the following groups: inter-plate earthquakes (occurring in the contact zone between the Nazca Plate and South American Plate), intermediate depth and large depth intra-plate earthquakes (occurring within the interior of the Nazca Plate), and shallow intra-plate earthquakes (occurring in the continental crust of the South American Plate). From the late 16th century to the present, there has been a high-magnitude earthquake every 8 to 10 years on average, throughout the Chilean territory.

Taking into account the particular characteristics of this project, such as the structure, height, rigidity, incorporation of base isolation, and also the requirements present in the national seismic design standards, it was necessary to perform a study based on the seismic hazards for the project site. The objective of this study was to establish a site specific design spectrum, according to the site's geographical location, geotechnical and geomorphic characteristics. To characterize the different seismic sources, earthquakes with a magnitude greater than 4 ( $M \geq 4$ ) and an epicenter within a radius of less than 300 km from the site (Fig. 5a) were considered. This allows the estimation of distances from the site to different seismogenic sources (Fig. 5b). The possible influence of shallow sources associated with the San Ramon Fault, located approximately 8.5 km from the site, was also evaluated (Fig. 5c). Even though there is uncertainty about its activity, accelerations were estimated as a benchmark for analyzing its possible influence on the overall seismic hazard of the site.

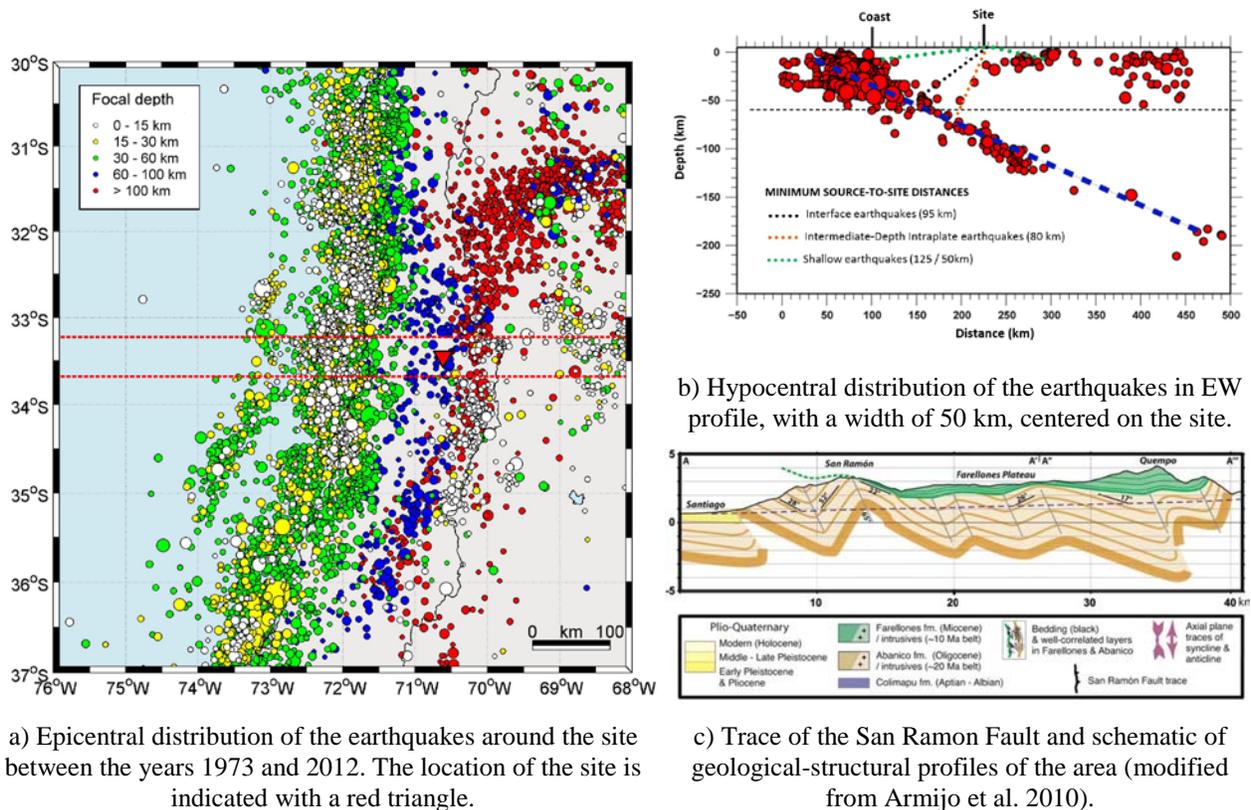


Fig. 5 - Seismic Hazard Study

Fig. 6a shows the spectral accelerations of the different seismic sources that could affect the site. In Fig. 6b, a comparison of the resulting acceleration spectra for two distinct earthquakes occurring at the San Ramon Fault is presented.



From the results of the seismic hazard study, the design elastic acceleration spectrum was determined for structures with periods up to seven seconds. Additionally, in accordance with the standards for seismic isolation design, seven pairs of artificial acceleration records, compatible with the proposed design spectrum were generated.

### 3. Seismic Isolation System

The seismic isolators of the building are located under the ends of the walls and under the columns. The project uses 24 natural rubber isolators, manufactured by Dynamic Isolation Systems. These devices are made of rubber with a strain capacity over 600%, whose long term properties are extremely stable. The design process considered an extensive database of experimental results obtained for similar isolators, subjected to comparable vertical loads, and subjected to even larger displacement levels. Of the 24 devices, 16 of them have a lead core (LRB), while the 8 remaining do not have a lead core (RB). The largest diameter Type C (RB) isolators are 155 cm in diameter and have a load capacity of more than 40,000 kN and will be located in the most heavily loaded locations under the ends of the walls of the elevator shafts. There are 8 Type A LRB isolators, 115 cm in diameter, with a load capacity of more than 20,000 kN, and 8 Type B isolators, 135 cm in diameter, with a load capacity of more than 30,000 kN. The stiffest isolators are the Type B isolators and they are located at the perimeter to control the torsion of the structure. The seismic isolation system achieves an effective vibration period near to 5 seconds, and about 20% effective damping. Based on the results of the seismic analyses, reductions of shears, absolute accelerations and interstory drifts by 70% to 80% are obtained compared to the fixed base elastic response.

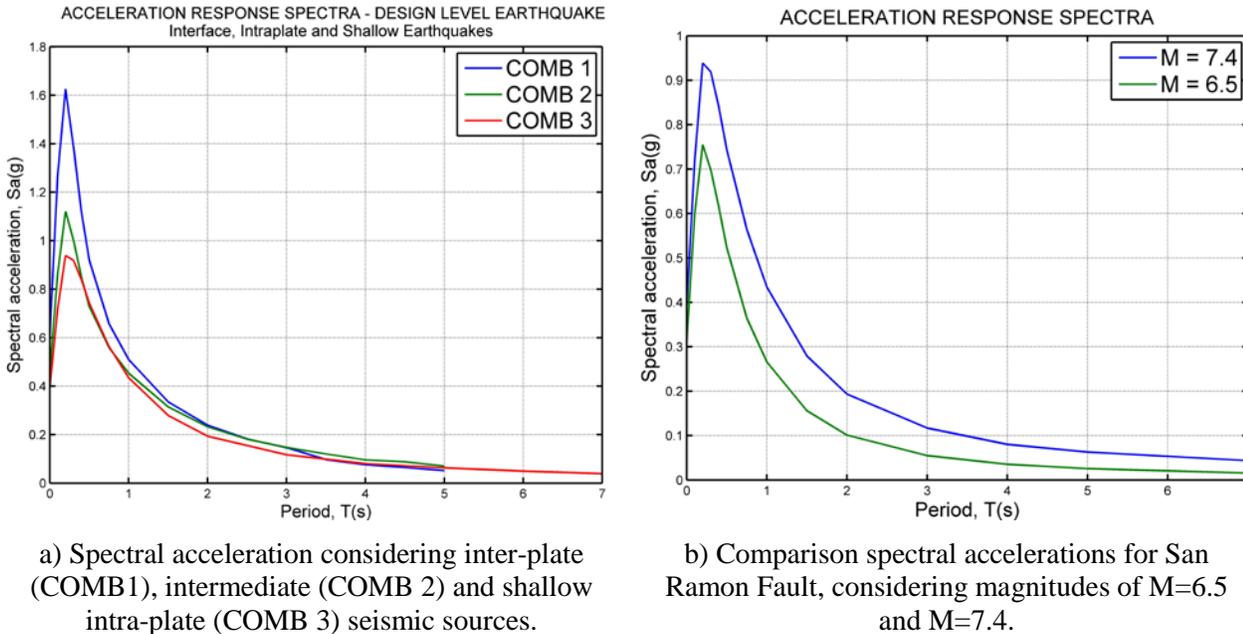


Fig. 6 - Site spectra

The seismic isolation system was subjected to an extensive test series carried out at Dynamic Isolation Systems' laboratories in McCarran, Nevada, and Eucentre laboratories in Pavia. Prototypes and all production isolators were subjected to an exhaustive test sequence of combined compression loads and shear displacements in order to validate the properties assumed for the design and to verify the stability of the isolators under extreme seismic loads.



## 4. Models, Analysis and Design of Superstructure

The Chilean isolation code allows for using either linear or nonlinear models to analyse and design the structure and the seismic isolation system. It also specifies two levels of seismic demand: the design basis earthquake and the maximum considered earthquake. The design basis earthquake (DBE) is used to analyse and design the whole structure except the seismic isolation system. On the other hand, the maximum considered earthquake (MCE) is used to analyse and design the seismic isolation system.

In the Nunoa Capital Building, two types of analyses were used. First, a linear analysis, specifically a response spectrum analysis, was performed. In this analysis, the isolators were modelled with their equivalent secant stiffness at the design displacement level. A site specific design spectrum obtained from the Seismic Hazard Evaluation was used. This spectrum represents the demand for the DBE level and was reduced in half for periods larger than 3.0 seconds to consider the extra damping provided by the seismic isolation system. This initial estimated factor was later verified through nonlinear analysis. This linear analysis was used to design the superstructure, the foundations and to define the seismic isolation system.

A nonlinear time history analysis was then carried out. In this analysis the isolators were modelled with a nonlinear biaxial hysteretic model that uses coupled plasticity properties for the two shear deformations [2]. This causes the extra damping of the seismic isolation system to be directly considered and provides an estimation of the effective damping of the system. This analysis was used primarily to validate the preliminary design of the seismic isolation system. Seven pairs of ground acceleration records obtained from the Seismic Hazard Evaluation were used. These ground acceleration records represent the demand for the MCE level. As allowed by the Chilean isolation code, all the parameters of interest such as displacements or axial loads on the isolators were obtained as the average of the peak response for each individual pair of records. As explained later, the superstructure is expected to remain essentially elastic for both the DBE and the MCE, which allows for modelling the building by using the linear elastic elements available in ETABS [2].

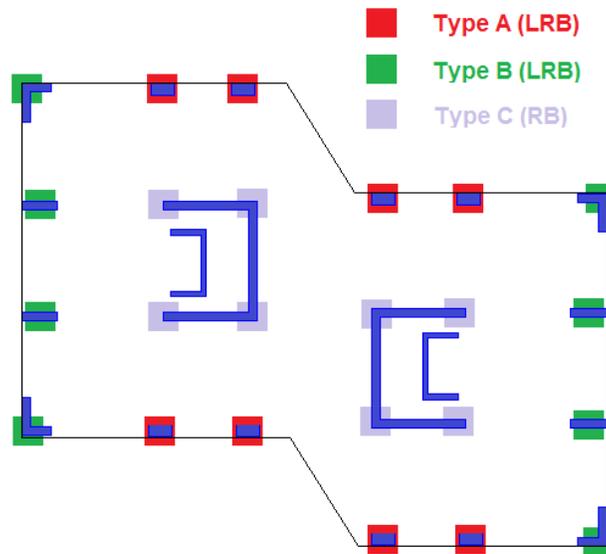


Fig. 7 - Isolator Layout

### 4.1 Global Behavior

The first eight vibration periods of the structure, for the design earthquake level, are shown in Table 1. The X direction refers to the long side and the Y direction to the short side of the structure. The Chilean isolation code allows for considering the design base shear as the elastic demand divided by a response modification factor  $R = 2$ , but no less than a minimum percentage of the seismic weight that depends on the seismic zone, specifically 5% for this project. Usually, the minimum base shear is between the elastic and the reduced demand, which causes the effective reduction factor  $R^*$  (the adjusted R factor to achieve the minimum base shear) to be between



1 and 2. Notwithstanding this, ~~the~~ the elastic demand is less than the minimum due to the long periods achieved in this project. This causes the superstructure to be designed for a seismic demand greater than the elastic demand, which is equivalent to have an effective reduction factor  $R^*$  of less than 1. The elastic demands, effective reduction factors  $R^*$ , final design shears for each direction, and other key design parameters, are shown in Table 2.

Table 1 – Periods of the Structure for the design earthquake level.

Mode	Period (s)	Dir X Mass Ratio (%)	Dir Y Mass Ratio (%)	Rot. Mass Ratio (%)
1	5.34	98.25	0.56	0.04
2	5.26	0.56	98.61	0.00
3	4.83	0.04	0.00	98.65
4	2.13	0.00	0.00	0.05
5	1.55	0.03	0.53	0.00
6	1.41	1.10	0.01	0.01
7	1.40	0.01	0.00	0.68
8	1.13	0.00	0.28	0.00

Table 2 – Elastic and Final Design Base Shear.

	X Direction	Y Direction
Elastic Base Shear (kN)	13,368	13,734
Design Base Shear $R=2$ (kN)	6,684	6,867
Minimum Design Base Shear (kN)	20,346	20,346
Effective Reduction Factor $R^*$	0.657	0.675
Final Design Base Shear (kN)	20,346	20,346



Table 3 – Contribution of each vibration mode to the base overturning moment.

	Mode	Base Shear kN	Base Overturning Moment, X Direction kN-m	Base Overturning Moment, Y Direction %
Isolated Structure	ALL MODES (CQC)	20,346	1,043,967	100.0
	MODE 1	19,955	837,546	80.2
	MODE 2	117	4,861	0.47
	MODE 3	11	376	0.04
	MODE 4	0	0	0.00
	MODE 5	61	10,334	0.99
	MODE 6	2,906	607,879	58.2
	MODE 7	30	6,257	0.60
	MODE 8	1	291	0.03
Fixed Base Structure	ALL MODES (CQC)	20,346	868,339	100.0
	MODE 1	13,950	835,313	96.2
	MODE 2	395	23,367	2.69
	MODE 3	43	2,529	0.29
	MODE 4	4	236	0.03
	MODE 5	13	587	0.07
	MODE 6	1	42	0.00
	MODE 7	8	74	0.01
	MODE 8	43	206	0.02

The base overturning moments in the isolated structure (for the final base design shear) in the X direction is amplified when compared to the fixed base structure solution. (Table 3).

#### 4.2 Design Challenges

As explained previously, for this particular project there were two characteristics that are not common for low rise isolated structures: a design base shear greater than the elastic demand, and an increased design base overturning moment. Those two characteristics imposed two big challenges for the structure design: to control the interstory drift and to avoid tension forces on the isolators.

The Chilean isolation code limits the story drifts to be less than 0.25% for the DBE level. This is different to other codes such as the ASCE7 [3] because it is directly measured from the reduced (by the R\* factor) DBE spectrum instead of multiplying the elastic response by Cd/I.

In order to control the story drifts, it was necessary to stiffen the building, which was a challenge because a tight architectural plan with almost no space for additional or thicker structure. This was achieved by using 2 outriggers per tower in the upper (mechanical) floor (Fig. 9). Even though this location was not optimal, the outrigger's benefits were enough to stiffen the structure and reduce the interstory drifts to below the code limits. This can be seen for the X direction in the Fig. 8.

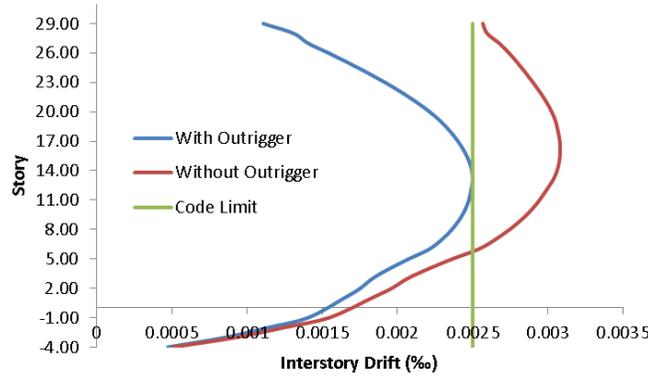


Fig. 8 - Interstory Drift for the DBE with and without the Outrigger.

The second challenge was to avoid tension forces on the isolator. Although the isolators can resist small tension forces, the design team decided to eliminate tension forces on the isolators for the MCE level. This was achieved by connecting the slabs of the two towers at the underground levels and by using a 2 m in thickness slab resting directly above the isolators (Fig. 10). This thick slab stiffens the isolated interface so that the external isolators are coupled with the internal ones, leading to a greater lever arm that resists the overturning moment and reduces the magnitude of the seismic compression/tension forces. Final compression forces on the isolators for the MCE level are shown in Fig. 911. To enhance the performance of this thick coupling slab, imbedded beams with closely spaced stirrups and ties were designed.

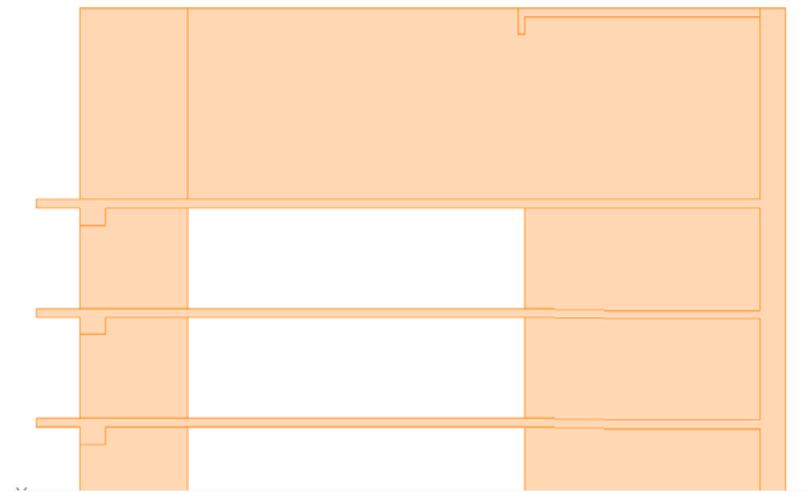


Fig. 9 – Elevation view showing roof outriggers

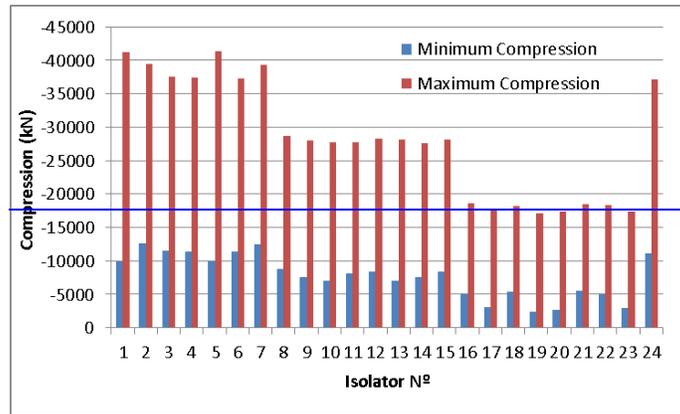


Fig. 9 – Compression Forces on each Isolator for the MCE.

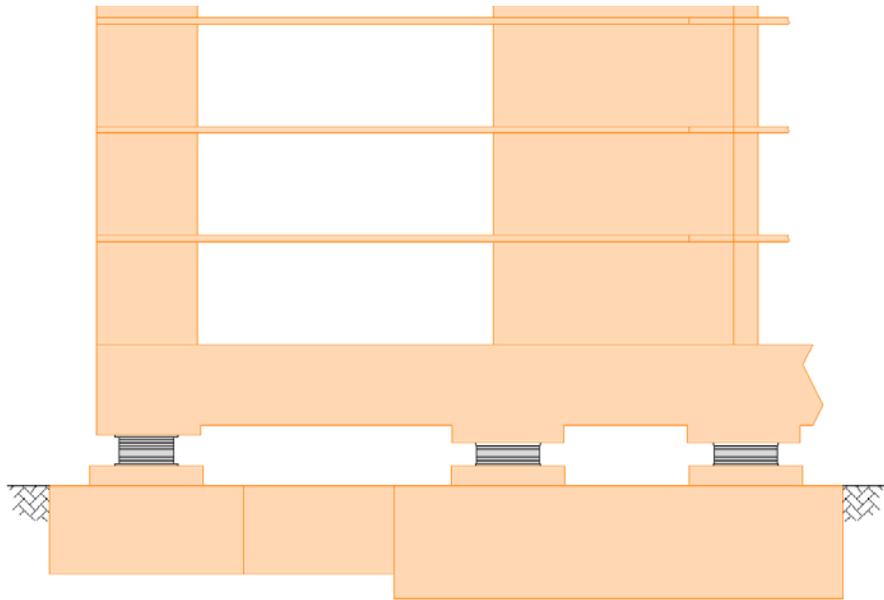


Fig. 10 – Elevation view of foundation, isolators, 2m thick slab and part of superstructure

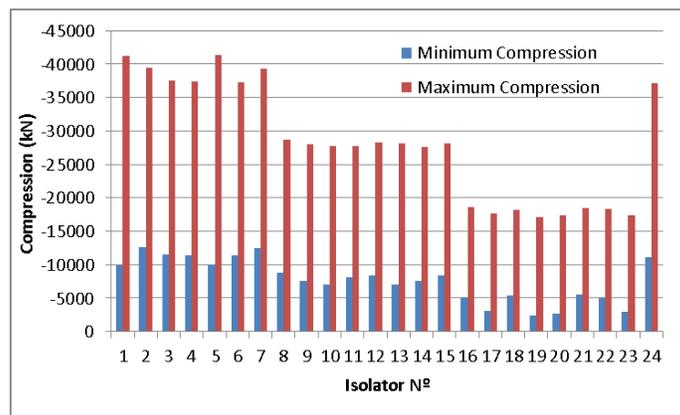


Fig. 11 - Compression Forces on each Isolator for the MCE.



## 5. Conclusions

The final design demonstrates the technical feasibility of implementing seismic isolation systems in high rise buildings. The main design challenges included the use of large diameter isolators to resist large compression forces, the difficulties to achieve simultaneously a high vertical stiffness and lateral flexibility, avoiding tension forces on the isolators and controlling the interstory drifts to a level below the code limits. The feasibility of using a seismic isolation system in this 33 story structure is associated to the good geotechnical conditions at the site, the use of a relatively stiff superstructure, and the use of an outrigger system. The sizes of the structural elements were determined by the interstory drifts limits rather than the design forces. It has been proven that the use of seismic isolation systems can considerably reduce the seismic demands, even in tall buildings such as the Nunoa Capital Building, and in consequence expecting less structural damage (elastic response of superstructure), nonstructural damage (reduced interstory drifts) and content losses (lower absolute accelerations) than its fixed base counterpart for an MCE level earthquake.

## 6. Acknowledgments

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